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LAMINAR CRACKING IN DAVIS BESSE NPS SHIELD BUILDING

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ABSTRACT

During the removal of concrete in the Shield Building for the Reactor Vessel Closure Head (RVCH) replacement at Davis-Besse nuclear plant, laminar cracking was observed in the plane of the outer reinforcement mat consisting of No. 11 bars. A detailed condition assessment was carried out to determine the extent of cracking. This evaluation indicated that cracking is present in most, if not all, of the architectural flute shoulders (Figure 1), two steam line penetration areas and in about 70% (in varying degrees) of the top 20 ft of the Shield Building cylinder. A detailed technical evaluation was also carried out to determine the effect of laminar cracking on the structural capacity of the Shield Building to resist its design basis loads. This technical evaluation indicated that effect of laminar cracking on the structural capacity of the Shield Building to resist its design basis loads should be minimal, if any. Two technical experts were retained to provide an independent opinion on the effect of laminar cracking on the structural capacity of the Shield Building. The technical assessment indicated that the only issue to be confirmed would be the effect of laminar cracking on the splices of No. 11 bars especially for the outer hoop reinforcement. Accordingly, two independent testing programs were carried out to evaluate the effect of laminar cracking on reinforcement bond in the splice regions. In one test program, the laminar crack was simulated by preloading the full-scale beam with No. 11 splices to yield and cracking the concrete and reloading the beam to failure. In the other test program, a cold joint was formed at the splice level by casting concrete in two separate layers and then loading to cause a crack, unloading and reloading to failure. From these independent test programs carried out at two different labs involving two industry experts it was concluded that No. 11 bars with a crack in the plane of the bars will develop their full yield capacity. The findings of these tests confirmed that there was no adverse effect of laminar cracking on the expected structural performance of the Shield Building.

INTRODUCTION

Cracking occurs in virtually all concrete structures and because of concrete's inherently low tensile strength can never be totally eliminated. Cracks can occur due to causes such as shrinkage, thermal or other load related situations. Cracks, if sufficiently large, can indicate structural problems; provide an open path to the environment for corrosion of the reinforcing steel; and inhibit a structure from meeting its performance requirements. Control of such cracking due to loads or imposed deformations is generally addressed through the American Concrete Institute (ACI 318) code requirements by specifying minimum reinforcing steel size and spacing including minimum bonded steel reinforcement and distribution of steel reinforcement.

Typically, cracks need to be repaired if they reduce the strength, stiffness, or durability of the structure to an unacceptable level, or if the function of the structure is seriously impaired. In addition, repairs that improve the appearance of the surface of a concrete structure may be desired from an aesthetic standpoint. Observations such as spalling, exposed reinforcement, and rust staining can be an indication that the reinforced concrete structure has deteriorated. Concrete that has been exposed to carbon dioxide from the environment can be identified by performing a carbonation test. This test can be used to assess the aging effect of the concrete as it is exposed to the environment.

The force transfer between the reinforcement steel and concrete is well established in ACI 408R-03. Per this document, the transfer of forces from the reinforcing steel to the surrounding concrete occurs by a combination of chemical adhesion, frictional at the interface and mechanical anchorage/bearing of deformed lugs (ribs in the reinforcement), (Figure 2). After the initial slip between the concrete and reinforcing steel, most of the force is transferred by mechanical bearing between the reinforcing steel ribs and the concrete. Given the rib dimensions of No. 10 or a No. 11 bar (0.064 and 0.071 in height for ASTM A615 steel), it is reasonable to assume that there is no loss of this mechanical bearing between the reinforcing steel and concrete in case of a tight crack (that is less than the rib height). ACI 408R-03 also indicates that friction also plays an important role in load transfer. This friction will not be affected as long as the crack is relatively tight and there is sufficient confinement.

STRUCTURAL EVALUATION

From observations and examinations performed it was concluded that cracks identified in the Shield Building do not adversely affect the structure from performing its intended function because of the following reasons:

- The laminar cracks are located adjacent to the reinforcing steel (hoop and vertical steel) in the shell, potentially creating a separation between the cylindrical shell and the architectural shoulders which were poured monolithically during construction. The shoulders were not considered as structural components in resisting design basis loads but serve as additional cover for the reinforcement.
- The laminar cracks identified through Impulse Response (IR) and core bores are tight and of hairline nature (mostly within 0.01 in with one crack 0.013 in). Surface inspections and observations do not indicate a separation of the flute shoulder from the shell. No surface rust stains exist on the outside surface of the Shield Building, again indicating that cracks are tight and stable (i.e. not active). The steel reinforcing was observed to have generally light corrosion or no corrosion at all.
- The #8 reinforcing steel tie bar; spaced 12 inches on centers tie the flute shoulder to the cylindrical shell and provides substantial confinement for the outer shell reinforcement. These horizontal reinforcing tie bars “anchor” the shoulders to the cylindrical shell limiting the width of the laminar crack. This reinforcing provides adequate confinement of the shell’s outside face reinforcement.
- For #10’s and #11’s reinforcing steel the required minimum height of the deformation is 0.064” and 0.071” respectively. Since the laminar crack widths are tight (generally ≤ 0.01”), this crack will not adversely affect the load transfer between the steel and concrete.
- Preliminary evaluation indicated that maximum stress in reinforcement at the critical section due to the controlling design basis loads is only 40% (a margin of 2.5 which indicates that actual expected bond stress demand will be significantly less. Furthermore, strength of in-place concrete is much higher (~6000 psi) than specified strength of 4000 psi at 28-days. Should the crack in any way, reduce the load transfer mechanism, its effect on bar-to-bar continuity within the structure will not be impacted since rebar stresses are sufficiently low.
- Note that in case of the Shield Building confinement for the vertical reinforcement in this case is provided by both the horizontal shell reinforcement as well as the No 8 bars connecting the flute shoulder to the shell (Fig. 1). Note that any separation or relative movement at the crack interface will engage the No. 8 bars in tension providing a clamping force to limit the crack width.

- With a tight crack (generally < 0.01 in with one crack 0.013 in) ensuring reinforcement bar interlock, the # 8 bars crossing the potential crack plane (connecting the flute shoulder to the shell), flute shoulders provide enough confinement for both vertical and horizontal reinforcement. For concrete outside the shoulders, visual observations indicated no spalls, popouts or staining of concrete. The IR sounding of concrete did not reveal any indications of loose concrete on the surface. Based on this, it is concluded that outside concrete cover is well bonded with the reinforcement.
- The flute shoulders serve as additional concrete cover to some of the main shell reinforcement. The cracks inside and outside the shoulder regions are all tight (generally < 0.01 in with one crack 0.013 in) and do not seem to have a path to the surface for air/moisture migration. Cracks are believed to have existed for a number of years within the thickness of the concrete in the flute shoulder regions without any significant corrosion. Based on this, there is no reason to believe that reinforcement has in any way lost its protection against environmental corrosion.
- Concrete tests conducted on the Shield Building indicated normal carbonation on the surface of the shell and almost negligible carbonation on the interface of the cracks indicating that the concrete crack face examined must not have been exposed to significant amounts of ambient environment (e.g. air). Tests indicated a carbonation depth of 5-8mm on the surface and minor carbonation on the fracture surface. These tests indicate that the cracking may have occurred after the initial and final set of the concrete. Tests also indicated that chloride content of concrete in the Shield Building is insignificant. The highest chloride content value measured was 0.090% (acid-soluble) and 0.037% (water-soluble). Tests also indicated that the concrete was in good condition.
- Comparing the carbonation of the concrete exterior surface with the cracked concrete surface could provide insights to the age of the crack. Since the cracks are tight and negligible carbonation on the cracked surface the age of the crack could not be accurately established. There is no evidence of significant corrosion on the reinforcement. It is therefore possible that cracks may have existed within the thickness of the concrete in the flute shoulder regions for some period of time. Note that there is no indication of surface cracking significant enough to provide a path for air/moisture to penetrate to the crack surface of interest. Also, where reinforcement bar has been exposed by the investigation there was no evidence of excessive corrosion indicating that cracking is not exposed to the environment.

TESTING PROGRAM

The initial technical evaluation discussed in Section 4 and 5 identified that the only issue of any structural significance was the possibility of having lap splices in the areas of laminar cracking. With cracking identified in most shoulder regions, two steam line penetrations areas and in top 20 ft of the Shield Building outside the Shoulder region, it is possible to have the following lap splices in cracked regions:

1. 79 in lap splices for No. 11 vertical bars.
2. 79 in laps for the hoops below El 780 which can fall in shoulder regions or in steam line penetration areas.
3. 120 in lap splices for hoop reinforcement above El 780 which can fall in shoulder regions or outside shoulder regions.

Note that No. 11 vertical bars are well confined by the outer hoops and additional concrete cover, especially in the shoulder regions. Therefore, splices for No. 11 vertical bars are not of as much significance as the outer No. 11 hoops. However, the test program addresses both 79 inch and 120 inch splices with minimum cover of 3-5 inches to cover all the possible situations identified above.

To investigate the effect of laminar cracking on force transfer capacity of No. 11 bars in splice regions, the following test program was established to cover the above mentioned worse case situations:

A: Testing of 79 in lap splice for No. 11 bars with 3-5 in cover

B: Testing of 120 in lap splice for No. 11 bars with 3-5 in cover

In order to ensure a reliable set of results, two independent and well known industry experts, Prof. Mete Sozen of Purdue University and Prof. David Darwin of University of Kansas were engaged to carry out a series of tests independently. The test program involved the following:

Purdue test program (see Table 1 and Figure 3) involved 6 beams with 79 inch splices and 6 beams with 120 in lap splices. In order to simulate laminar cracking in the plane of the bars, the splices were placed at 6 in spacing with a side cover of 3 inches. The laminar crack of 0.01 inches or more was initiated with a prior loading of up to yield and subsequent unloading (Figure 4).

Kansas test program (see Table 2) involved 3 beams of 79 in splices and 3 beams of 120 in splices. The first beam with 79 in splice was cast monolithically as a benchmark. In order to simulate laminar cracking in the plane of the bars, the remaining 5 beams were cast in two lifts one up to the center of the bars and the second pour the next day to the complete the casting to top of the beam. This process allowed formation of a standard cold joint in the plane of the bars which would serve as a weak plane and help simulate/produce a laminar crack during testing. The reinforcement cover of 3 in was maintained both on the sides and to the top surface of the beam. The laminar crack of 0.01 inches or larger was initiated in the specimen by prior loading and subsequent unloading.

Both Purdue and Kansas beams involved 2 splices side by side with 6 inches of spacing which presents a rather aggressive condition to give lower bound capacity results. Note that splices in the Shield Building are actually staggered with spacing of at least 12 to 24 inches. Also, the splices in the Shield Building conform to the curvature of the building which would provide additional confinement effect not included in the straight beam tests.

In both test programs at Purdue and Kansas, an effort was made to simulate the concrete mix of the Shield Building to the extent possible. Purdue used a similar mix and aggregate size. Since it was practically impossible to exactly match the concrete given the age of the plant, every effort was made to test at relatively lower (conservative) compressive strength and tensile strength values to produce conservative bond capacity values. Note that compressive and more importantly, tensile strength of concrete are recognized to be the key parameters of influence for bond strength of reinforcement in concrete. Moreover, Kansas tests were carried out at an age of only 7 days which resulted in lower bound compressive and tensile strengths thus giving very conservative or lower-bound results.

The average 28 day compressive strength from original construction of the Shield Building was 5836 psi. The average compressive strength of in-place concrete tested using cores taken during the Shield Building evaluation in 2011 was 7571 psi. The corresponding tensile strength of in-place concrete was determined to be 918 psi.

Figures 5 and 6 show results of testing at Purdue University and confirm that No. 11 bars with a crack in the plane of the bars will be able to develop their full yield both for 79 in splice as well as 120 in splice. This is despite the fact that beams were pre-cracked with 1st cycle loading and had splices next to each other (not staggered) at only 6 in spacing.

Table 3 and Figure 7 show results of testing at University of Kansas and confirm that No. 11 bars with a simulated crack (involving a cold joint that is pre-cracked, see Figure 8) will be able to develop near yield (57 and 62 ksi) for 79 in splice and full yield for 120 in splice. But it should be noted that these results are based on a very aggressive test condition of splices next to each other at 6 in spacing compared to that of the Shield Building where splices are staggered and in most cases placed at approximately 12 to 24 in apart.

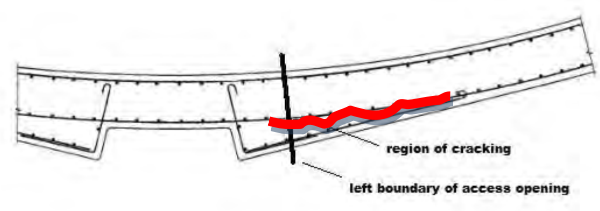

Figure 1. Cracking in Flute shoulder Region to the Left of the Construction Opening

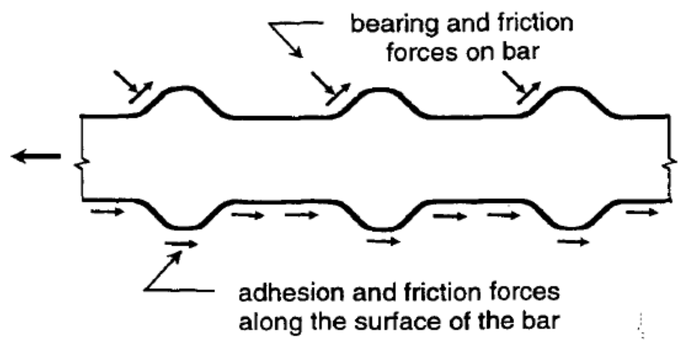

Figure 2 Bond Force Transfer

Table 1 Beams Tested at Purdue University

Test Girder Designation	Cast	Tested	Concrete Compressive Strength psi	Concrete Splitting Cylinder Strength psi	Lap Length in.
A1	17 April 2012	4 June 2012	5270	480	120
A2	17 April 2012	1 June 2012	6030	500	120
A3	17 April 2012	30 May 2012	5890	480	120
A4	24 April 2012	8 June 2012	5110	440	120
A5	24 April 2012	7 June 2012	5240	440	120
A6	24 April 2012	5 June 2012	5490	450	120
B1	10 April 2012	10 May 2012	4460	450	79
B2	10 April 2012	23 May 2012	4800	480	79
B3	10 April 2012	21 May 2012	4780	420	79
B4	30 April 2012	14 May 2012	5460	490	79
B5	30 April 2012	17 May 2012	5260	480	79
B6	30 April 2012	25 May 2012	5230	450	79

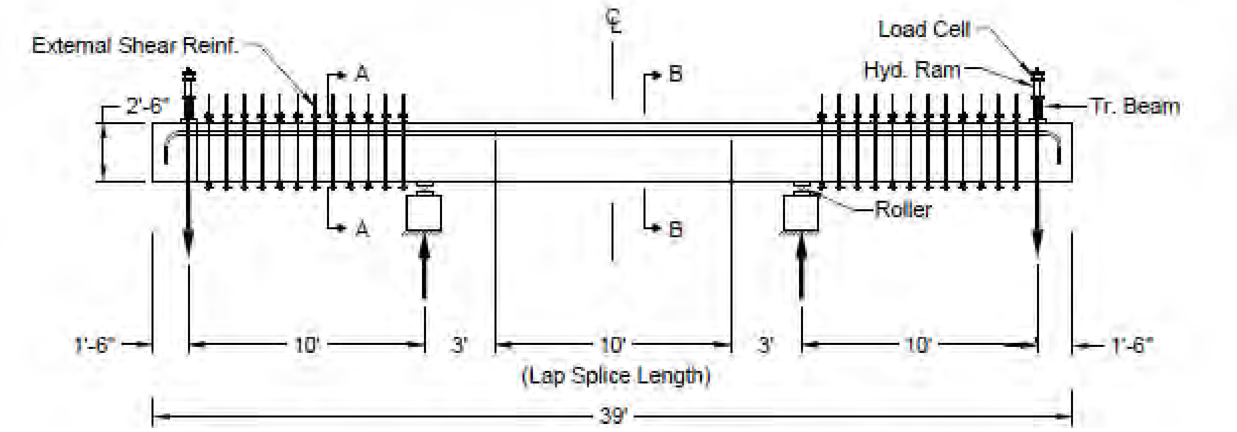

Figure 3 Test Girder, Series A Purdue Tests (120 in splice)

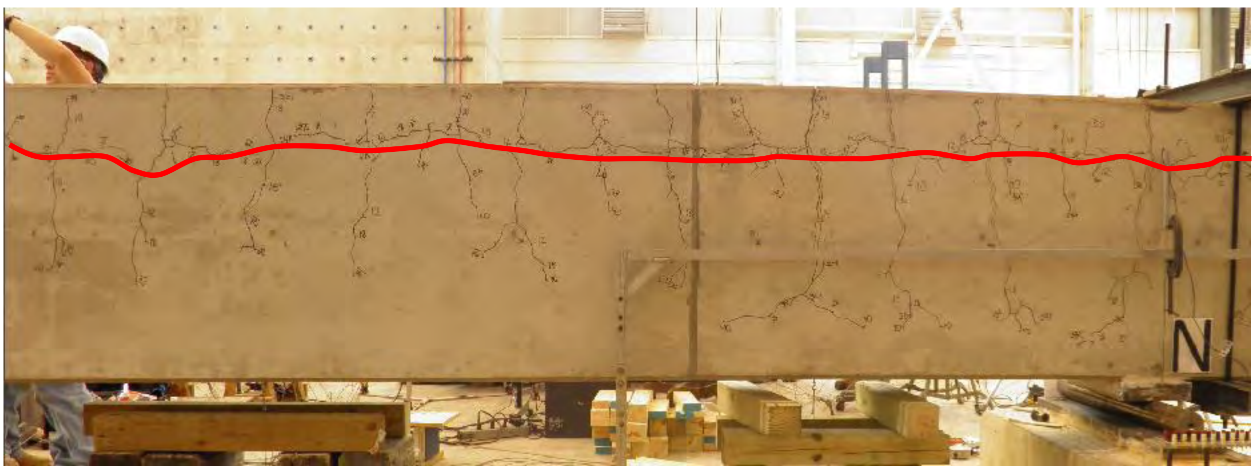

Figure 4 Typical Pattern of Flexural Cracks and Bursting Cracks Highlighted Resulting in Laminar Cracking (Purdue Testing)

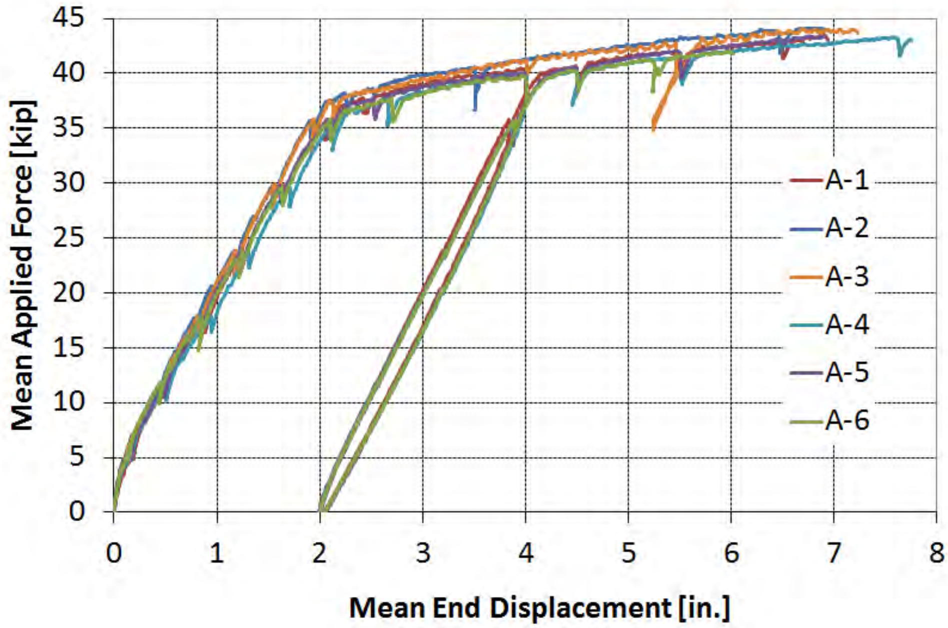

Figure 5 Force-Displacement for A-Series (120 in Laps) of Tests at Purdue

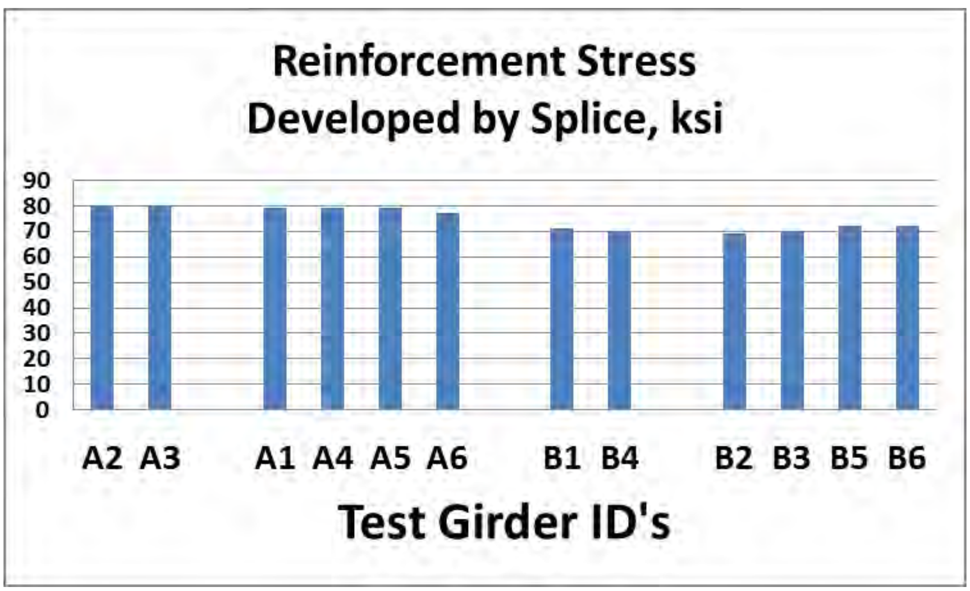

Figure 6 Maximum Unit Stress Reached by Lap Splices at Purdue

Table 2 Summary of Design Beam Dimensions for Beam-Splice Specimens at University of Kansas

Specimen	Splice length (in.)	Simulated crack	Nominal Beam dimensions					
			Support Spacing (ft)	Span, L (ft)	Width, b (in.)	Height, h (in.)	Effective Depth, d (in.)	Depth to A_s', d', (in.)
B1	79	None (monolithic)	11	25	18	24	20.3	2.8

B2	79	Cold joint	11	25	18	24	20.3	2.8
B3	79	Cold joint	11	25	18	24	20.3	2.8
B4	120	Cold joint	14	28	18	24	20.3	2.8
B5	120	Cold joint	14	28	18	24	20.3	2.8
B6	120	Cold joint	14	28	18	24	20.3	2.8

Table 3 Bar Stresses at Failure for Beam-Splice Specimens at University of Kansas

Beam ID – Splice length	Concrete strength, psi	Concrete cover, in.[a]	Total load at splice failure, kips	Calculated moment at splice failure, kip-ft	Calculated bar stress at failure, ksi		Failure mode
					Equiv. stress block	Moment-curvature	
1 – 79 in. (monolithic)	5330/ 4330[+]	3/3/3	103	344	70	70	Flexural Failure [*]
2 – 79 in. (cold joint, loaded monotonically)		3/3/3	85	292	59	62	Splice failure[**]
3 – 79 in. (cold joint, unloaded and reloaded)		3.25/3.35/2.9	80	270	53	57	Splice failure[**]
4 – 120 in. (cold joint, loaded monotonically)	5230/ 5490[+]	3/2.8/3.4	105	350	71	72	Splice failure and secondary flexural failure[***]
5 – 120 in. (cold joint, unloaded and reloaded)		3.15/3/15/3.15	96	325	66	67	Splice failure and secondary flexural failure[***]
6 – 120 in. (cold joint, unloaded and reloaded)		3.15/3.15/2.9	100	338	69	69	Splice failure and secondary flexural failure[***]

[a] Top cover/north side cover/south side cover
[+] Compressive strength of concrete below and above the cold joint.
[*] Test was stopped after reinforcing steel yielded, when crushing of the concrete in the compression zone was observed.
[**] Splice failed prior to yielding of the flexure reinforcement.
[***] Splice failed after yielding of the flexure reinforcement

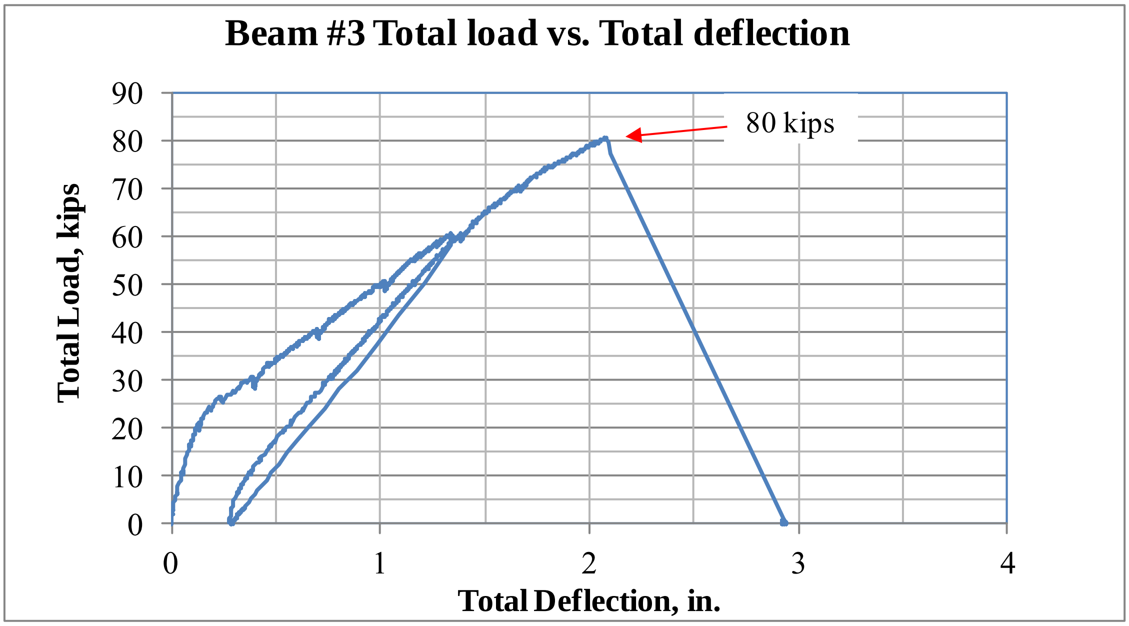

Figure 7 Total Load vs. Total Deflection for Beam 3 (with a cold joint) at University of Kansas

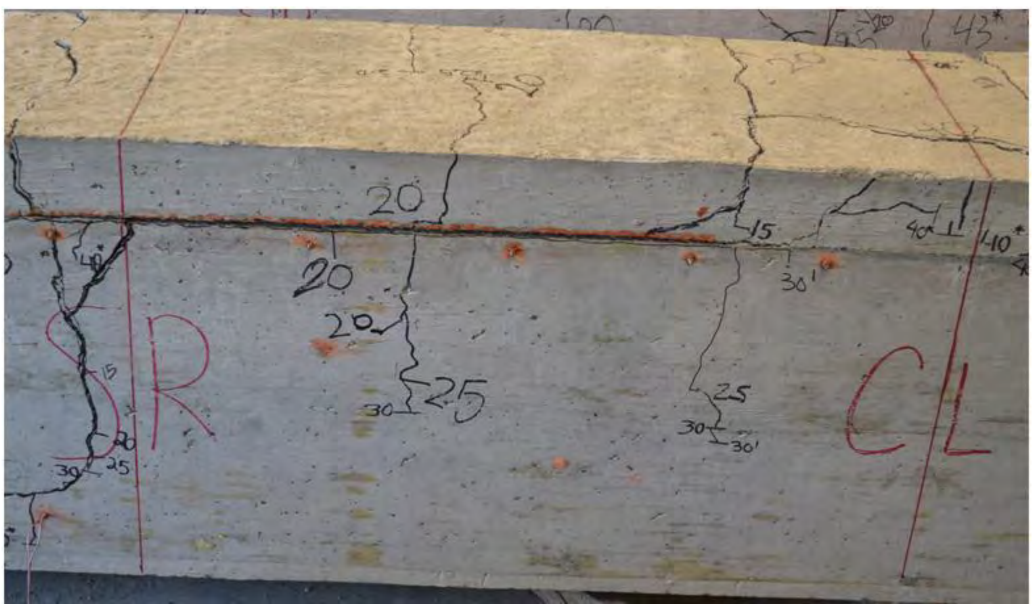

Figure 8 Beam 3 Failure with Wide Horizontal Cracks Along Cold Joint, University of Kansas

CONCLUSION

During the removal of concrete in the Shield Building for RVCH replacement at Davis-Besse Nuclear Power Plant, laminar cracking was observed in the plane of the outer reinforcement mat consisting of No. 11 bars. A detailed condition assessment was carried out to determine the extent of cracking as described in Section 3. This evaluation indicated that cracking is present in most, if not all, of the architectural flute shoulders, two steam line penetration areas and in about 70% of the top 20 ft of the Shield Building cylinder. A detailed technical evaluation was also carried out to determine the effect of laminar cracking on the structural capacity of the Shield Building to resist its design basis loads. This technical evaluation indicated that effect of laminar cracking on the structural capacity of the Shield Building to resist its design basis loads should be minimal, if any. Two outside experts were retained to provide an independent opinion on the effect of laminar cracking on the structural capacity of the Shield Building. Both experts agreed with the technical assessment but recommended that the only issue to be confirmed would be the effect of laminar cracking on the splices of No. 11 bars especially for the outer hoop reinforcement. Accordingly, tests were carried out at Purdue University and University of Kansas which involved the following:

Purdue test program involved 6 beams with 79 inch splices and 6 beams with 120 in lap splices. In order to simulate laminar cracking in the plane of the bars, the splices were placed at 6 in spacing with a side cover of 3 inches. The laminar crack of 0.01 inches or more was initiated with a prior loading of up to yield and subsequent unloading.

Kansas test program involved 3 beams of 79 in splices and 3 beams of 120 in splices. The first beam with 79 in splice was cast monolithically as a benchmark. In order to simulate laminar cracking in the plane of the bars, the remaining 5 beams were cast in two lifts one up to the center of the bars and the second pour the next day to the complete the casting to top of the beam. This process allowed formation of a standard cold joint in the plane of the bars which would serve as a weak plane and help simulate/produce a laminar crack during testing. The reinforcement cover of 3 in was maintained both on the sides and to the top surface of the beam. The laminar crack of 0.01 inches or larger was initiated in the specimen by prior loading and subsequent unloading.

Both Purdue and Kansas beams involved 2 splices side by side within 6 inches of spacing which presents a rather aggressive condition and likely to give lower bound capacity results. Note that splices in the Shield Building are actually staggered with spacing of at least 12 inches. Also, the splices in the Shield Building conform to the curvature of the building which would provide additional confinement effect not included in the straight beam tests.

The testing at Purdue confirmed that No. 11 bars with a crack in the plane of the bars will be able to develop their full yield both for 79 in splice as well as 120 in splice. This is despite the fact that beams were pre-cracked with 1st cycle loading and had splices next to each other (not staggered) at only 6 in spacing.

The testing at University of Kansas confirmed that No. 11 bars with a simulated crack (involving a cold joint that is pre-cracked) will be able to develop near yield (57 and 62 ksi) for 79 in splice and full yield for 120 in splice. But it should be noted that these results are based on a very aggressive test condition of splices next to each other at 6 in spacing compared to that of the Shield Building where splices are staggered and in most cases placed at approximately 12 to 24 in apart. The bars are expected to develop their full yield capacity given their actual configuration, layout and spacing and the actual strength of concrete in the Shield Building.

From the independent test programs carried out at two different labs involving two industry experts it is concluded that No. 11 bars with a crack in the plane of the bars present in the Shield Building will develop their full yield capacity. The findings of these tests confirmed that there was no adverse effect of laminar cracking on the expected structural performance of the Shield Building.

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